DAM-BREACH MODELING AND FLOOD ROUTING:
A PERSPECTIVE ON PRESENT CAPABILITIES AND FUTURE DIRECTIONS[1]

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Abstract: Dam-breach modeling and the associated routing of the breach unsteady outflow through the downstream river/valley is a continuing concern to many Federal, state, local agencies, etc., which are either charged with or assist those so charged with dam design, operation, regulation, and/or public safety. A brief historical summary is provided which covers many of the relevant procedures aimed at the prediction of dam-breach floods and their extent of flooding within the downstream river/valley.

Dam-breach modeling can be conveniently categorized as parametric-based or physically-based. The former utilizes key parameters: average breach width (b_{av}) and breach formation time(t_f) to represent the breach formation in earthen dams, and thus compute the breach outflow hydrograph using a numerical time-stepping solution procedure or a single analytical equation. Statistics on observed values for b_{av} and t_f have been presented. Also, various regression equations have been developed to compute the peak-breach discharge using only the reservoir volume (V_r) and the dam height (H_d) or some combination thereof. Physically-based breach models use principles of hydraulics, sediment erosion, and soil stability to construct time-stepping solutions of the breach formation process and the breach outflow hydrograph.

Flood routing is essential for assessing the extent of downstream flooding due to dam-breach outflows because of the extreme amount of peak attenuation that such unsteady flows experience during propagation through the downstream river/valley. Dam-breach flood routing models have utilized (1) numerical solutions of the complete one-dimensional Saint-Venant equations of unsteady flow; (2) breach peak-flow routing attenuation curves coupled with the Manning equation to compute peak-flow depths; and (3) a simplified Muskingum-Cunge flow routing and Manning equation depth computation. The latter two routing approaches incur additional error compared to the Saint-Venant routing. Some future research/development directions for dam-breach prediction capabilities are presented.

1. Introduction

A breach is the opening formed in a dam as it fails. The actual

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breach failure mechanics are not well understood for either earthen or concrete dams. Prior to about 1970, most attempts to predict downstream flooding due to dam failures, assumed that the dam failed completely and instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch dams with reservoir storage volumes greater than about one-half million acre-ft, but they are not appropriate for either earthen dams or concrete gravity dams.

Dam-break modeling and the associated routing of the outflow hydrograph (flood wave) through the downstream river/valley is of continuing concern to many Federal, state, local, and international agencies, the private sector, and academia. Such predictive capabilities are of concern to these entities since they are charged with or assist those charged with responsibilities for dam design, operation, regulation, and/or public safety. This paper presents a perspective on the present capabilities to accomplish dam-breach modeling and the associated flood routing.

Dam-breach models may be conveniently categorized as parametric models or physically-based models. A summary of a relevant portion of the history of dam-breach modeling and dam-breach flood routing capabilities is presented in Table 1. A brief description of both numerical and analytical dam-breach parametric models, dam-breach physically-based models, and dam-breach flood routing models are presented herein.

Finally, future research/development directions in dam-breach prediction capabilities are presented. These are judged to offer the most efficient and effective means of improving practical dam-breach modeling and dam-breach flood routing capabilities.

2.   Numerical Parametric Breach Models

Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width (b_{av}) in the range ($0.5 \leq b_{av}/H_d \leq 8$) where H_d is the height of the dam. The middle portion of the range for b_{av} is supported by the summary report of Johnson and Illes(1976) and the upper range by the report of Singh and Snorrason(1982). Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time (t_f) for its formation through erosion of the dam materials by the escaping water. The breach formation time is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is <u>after</u> the downstream face of the dam has eroded away

YEAR	DAM - BREACH OUTFLOW		Physical/ Numerical Erosion/Collapse/Hydraulics	DAM-BREACH Flood Routing
	Parametric			
	Numerical	Analytical		
1969-70	$Q=f(a,t_f, S_a, Q_i)$ Fread, Dissertation	$Q=f(B_r, a, t_f, H_d, S_a)$ Fread	----	----
1977	NWS DAMBRK Last version: 1991	----		NWS DAMBRK Last Version : 1991 1 - D
1981	----	$Q=3.1B_r \left(\dfrac{C}{t_f + C/\sqrt{H_d}}\right)^3$ $C=23.4\ S/H_d$	----	NWS SMPDBK Last Version:1991 Curves from DAMBRK
1981-88	----	$Q=f(H_d, V_R, ...)$ Hagen, McDonald, Evans, Costa 1982 1984 1986 1988	Ponce/Tsivoglu Erosion/Hydraulic 1981 1-D	----
1983-84	----	----	NWS BREACH Last Version: 1991	----
1987	---- Statictical Data on Parameters Froelich, 1987,1995		----	----
1988	----	Singh/Quiroga	BEED (Singh/Quiroga) 1-DSed (Macchione)	BEED (Mushingum-Cunge)
1993	----	----	2-D/Sed (Bechleler/Broich)	----
1995	NWS FLDWAV same as DAMBRK	----		NWS FLDWAV 1- D Multi-River

Table 1. History of Dam-Breach Modeling

and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face. This portion of the failure process could be thought of as the "initiation time" which is quite distinct from the breach "formation time" or time of failure (t_f). Time of failure (t_f) for overtopping initiated failures may be in the range of a few minutes to usually less than an hour, depending on the height of the dam, the type of materials, and the magnitude and duration of the overtopping flow of the escaping water.

Poorly constructed coal-waste dumps (dams) which impound water tend to fail within a few minutes, and have average breach widths in the upper range of the earthen dams mentioned above. Also, average breach widths are considerably larger for reservoirs with very large storages which sustain a fairly constant reservoir elevation during the breach formation time; such a slowly changing reservoir elevation enables the breach to erode to the bottom of the dam and then erode horizontally creating a wider breach before the peak discharge is attained.

Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by the escaping water. Breach formation times are usually considerably longer for piping than overtopping failures since the upstream face is slowly being eroded in the early phase of the piping development. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by caving-in of the top portion of the dam.

Fread(1971,1977,1988,1993) used a parametric approach to describe and mathematically model the dynamic breach-forming process. The mathematical model combined the reservoir level-pool routing equation with a critical-flow, weir equation in which the weir or breach was time dependent whose shape and size were controlled by specified parameters. The numerical time-stepping solution of these equations produced a discharge hydrograph of breach outflow including the maximum (peak) discharge. The parametric description of the dynamic breach is shown in Figure 1.
The breach is assumed to form over a finite interval of time (t_f) and has a final (terminal) breach size of b determined by the breach side-slope parameter (z) and the average breach width parameter b_{av}. Such a parametric representation of the breach is
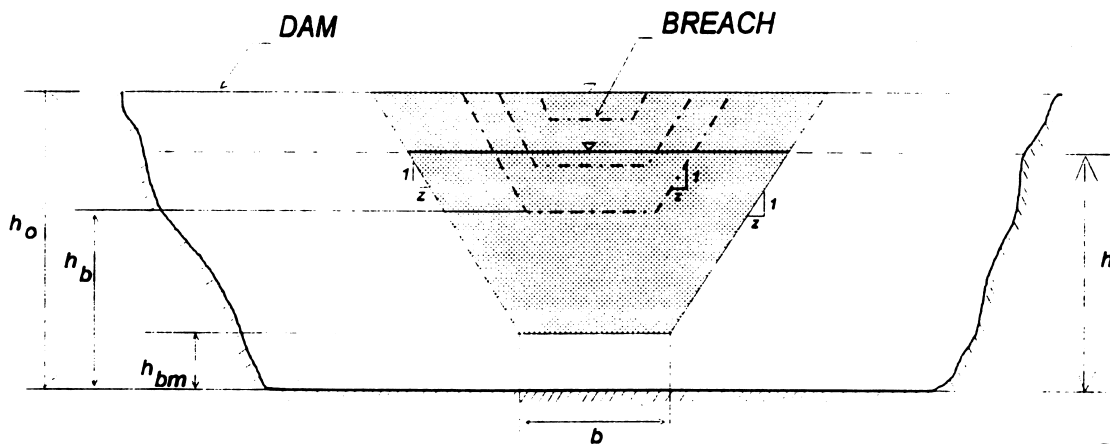

Figure 1 - Front View of Dam Showing Formation of Breach.

utilized for reasons of simplicity, generality, wide applicability, and the uncertainty in the actual failure mechanism. This approach to the breach description follows that first used by Fread and Harbaugh (1973) and later in the NWS DAMBRK Model (Fread;1977, 1988). The shape parameter (z) identifies the side slope of the breach, i.e., 1 vertical: z horizontal. The range of z values is from 0 to somewhat larger than unity. The value of z depends on the angle of repose of the compacted, wetted materials through which the breach develops. Rectangular, triangular, or trapezoidal shapes may be specified by using various combinations of values for z and the terminal breach bottom width (b), e.g., $z=0$ and $b>0$ produces a rectangle and $z>0$ and $b=0$ yields a triangular-shaped breach. The terminal width b is related to the average width of the breach (b_{av}) by the following:

$$b = b_{av} - zH_d \qquad (1)$$

in which H_d is the height of the dam. The bottom elevation of the breach (h_b) is simulated as a function of time (t_f) according to

the following:

$$h_b = h_d - (h_d - h_{bm})\ (t_b/t_f)^\rho \quad o \leq t_b \leq t_f \tag{2}$$

in which h_d is the elevation of the top of the dam. The model assumes the instantaneous breach bottom width (b_i) starts at a point (see Figure 1) and enlarges at a linear or nonlinear rate over the failure time (t_f) until the terminal bottom width (b) is attained and the breach bottom elevation (h_b) has eroded to a specified final elevation (h_{bm}). The final elevation of the breach bottom (h_{bm}) is usually, but not necessarily, the bottom of the reservoir or outlet channel bottom, t_b is the time since beginning of breach formation, and ρ is a parameter specifying the degree of nonlinearity, e.g., $\rho=1$ is a linear formation rate, while $\rho=2$ is a nonlinear quadratic rate; the range for ρ is $1 \leq \rho \geq 4$. The linear rate is usually assumed, although the nonlinear rate is more realistic especially for piping failures; however, its value is not well identified. The instantaneous bottom width (b_i) of the breach is given by the following:

$$b_i = b\ (t_b/t_f)^\rho \quad t_b \leq t_f \tag{3}$$

During the simulation of a dam failure, the actual breach formation commences when the reservoir water surface elevation (h) exceeds a specified value, h_f. This enables the simulation of an overtopping of a dam in which the breach does not form until a sufficient amount of water is flowing over the top of the dam. A piping failure may also be simulated by specifying the initial centerline elevation of the pipe-breach.

2.1 Statistically-Based Breach Predictors

Some statistically derived predictors for b_{av} and t, have been presented in the literature, i.e, MacDonald and Langridge-Monopolis(1984) and Froehlich(1987,1995). Using this data of the properties of 63 breaches of dams ranging in height from 15 to 285 ft, with 6 of the dams greater than 100 ft, the following predictive equations are obtained:

$$b_{av} = 9.5k_o (V_r\ H)^{0.25} \tag{4}$$

$$t_f = 0.3 V_r^{0.53}/H^{0.9} \tag{5}$$

in which b_{av} is average breach width (ft), t_f is time of failure (hrs), $k_o = 0.7$ for piping and 1.0 for overtopping, V_r is volume(acre-ft) and H is the height (ft) of water over the breach bottom (H is usually about the height of the dam, H_d). Standard error of estimate for b_{av} is ±56 percent of b_{av}, and the standard error of estimate for t is ±74 percent of τ.

3. Analytical Parametric Breach Models

A single analytical equation was also developed to predict the
peak outflow from a breached dam. Fread(1981,1984) developed
such an equation which was a critical component of the NWS SMPDBK
(**S**implified **D**am-**B**reak) model. This equation accounted for the
hydraulic processes of dam-breach outflows, i.e, the simultaneous
lowering of the reservoir elevation as the breach forms by the
escaping reservoir outflow while using the basic breach
parameters (b_{av}, t_f), i.e.

$$Q_p = 3.1 b_{av} [C/(t_f + C/H_d^{0.5})]^3 \qquad (6)$$

in which Q_p is the peak breach outflow in cfs, b_{av} is the average
breach width in feet, t_f is the breach formation time in hours,
H_d is the height of the dam in feet, and $C = 23.4\ S_a/b_{av}$ in which S_a
is the reservoir surface area (acres) somewhat above the
elevation of the top of the dam. Recently, a similar but
considerably more complicated approach was reported by Walder and
O'Connor(1997).

Another analytical (single equation) approach for _earthen_ dam-
breaches relies on a statistical regression approach that relates
the observed (estimated) peak dam-breach discharge to some
measure of the impounded reservoir water volume: depth, volume,
or some combination thereof, e.g., Hagen,1982; Evans,1986; Costa,
1988; Froehlich,1995. An example of this type of equation
follows:

$$Q_p = a\ v_r^b H_d^c \qquad (7)$$

in which V_r is the reservoir volume, H_d is the height of the dam,
and a, b, c are regression coefficients, e.g., Froehlich(1995)
quantifies these as $a=40.1$, $b=0.295$, $c=1.24$ in which the units
for Q_p, V_r, and H_d are cfs, acre-ft and ft, respectively. This
approach is expedient but generally only provides an order of
magnitude prediction of dam-breach peak discharge. It does not
reflect the true hydraulics, but instead mixes the failure-
erosion process and the hydraulic processes, while ignoring the
important components of time-dependent erosion, weir flow, and
reservoir routing.

4. Physically-Based Breach Erosion Models

Another means of determining the breach properties is the use of
physically-based breach erosion models. Cristofano(1965) modeled
the partial, time-dependent breach formation in earthen dams;
however, this procedure required critical assumptions and
specification of unknown critical parameter values. Also, Harris
and Wagner(1967) used a sediment transport relation to determine
the time for breach formation, but this procedure required
specification of breach size and shape in addition to two

critical parameters for the sediment transport computation; then, Ponce and Tsivoglou(1981) presented a rather computationally complex breach erosion model which coupled the Meyer-Peter and Muller sediment transport equation to the one-dimensional differential equations of unsteady flow (Saint-Venant equations) and the sediment conservation equation. They compared the model's predictions with observations of a breached landslide-formed dam on the Mantaro River in Peru. The results were substantially affected by the judicious selection of the breach channel hydraulic friction factor (Manning n), an empirical breach width-flow parameter, and an empirical coefficient in the sediment transport equation.

Another physically-based breach erosion model (BREACH) for earthen dams was developed (Fread;1984,1987) which utilizes principles of soil mechanics, hydraulics, and sediment transport. This model substantially differed from the previously mentioned models. It predicted the breach characteristics (size, shape, time of formation) and the discharge hydrograph emanating from a breached earthen dam which was man-made or naturally formed by a landslide; the typical scale and geometrical variances are illustrated in Figure 2. The model was developed by coupling the
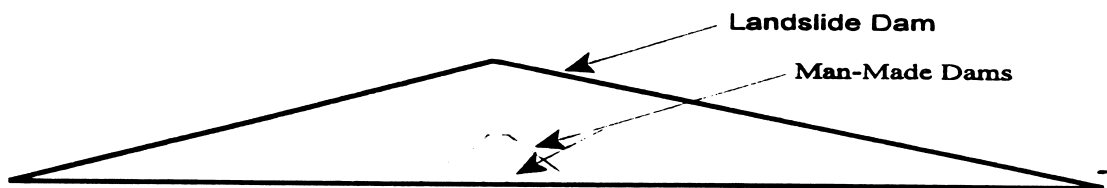

Figure 2. Comparative View of Natural Landslide Dams and Man-Made Dams.

conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity (computed along an erosion-formed breach channel. The bottom slope of the breach channel was assumed to be the downstream face of the dam as shown in Figure 3. The growth of the breach channel, conceptually modeled as shown in Figure 4, was dependent on the dam's material properties (D_{50} size, unit weight(γ), internal friction angle (ϕ), and cohesive strength (C_n)).

The model considered the possible existence of the following complexities: (1) core material properties which differ from those of the outer portions of the dam; (2) formation of an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water; (3) the downstream face of the dam could have a grass cover or be composed of a material such as rip-rap or cobble stones of larger grain size than the major portion of the dam; (4) enlargement of the breach through the mechanism of one or more sudden structural
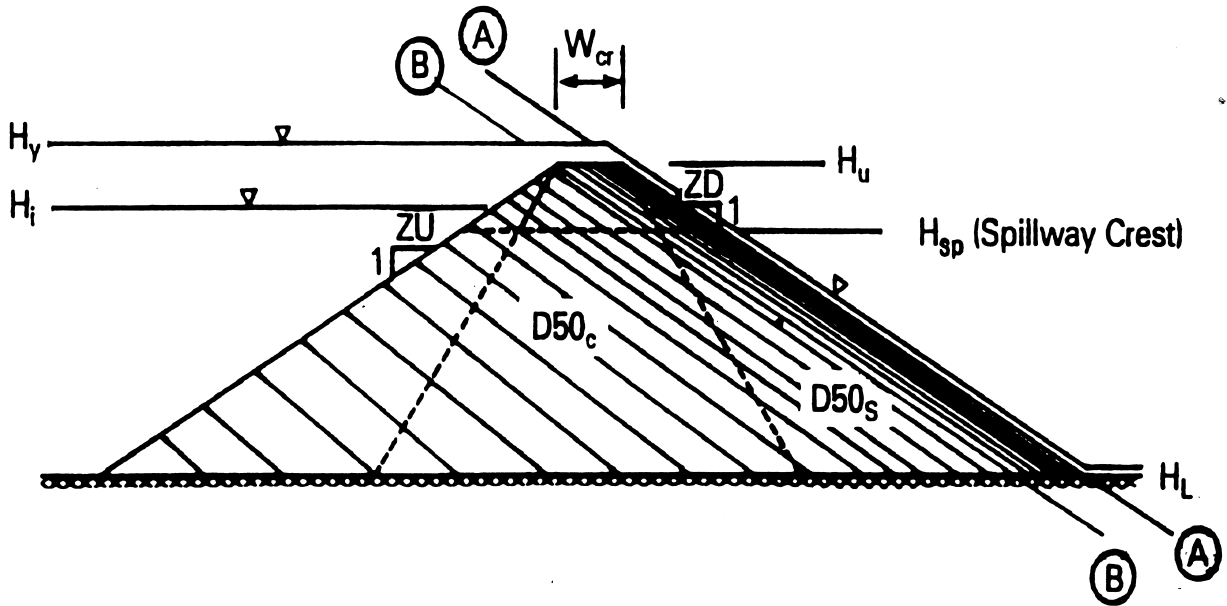
Figure 3. Side View of Dam Showing Conceptualized Overtopping Failure Sequence.

collapses of the breaching portion of the dam due to the
hydrostatic pressure force exceeding the resisting shear and
cohesive forces; (5) enlargement of the breach width by collapse
of the breach sides according to slope stability theory as shown
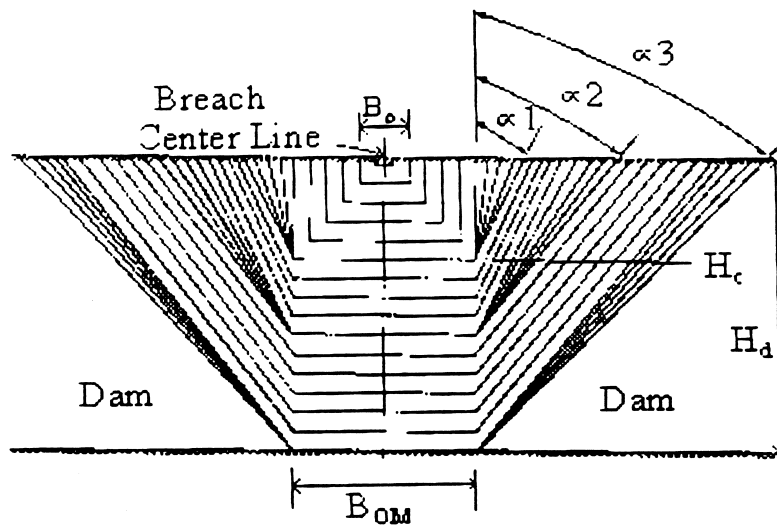in Figure 4; and (6) the capability for initiation of the breach

Figure 4. Front View of Dam with Breach Formation
Sequence.

via piping with subsequent progression to a free-surface breach
flow. The outflow hydrograph was obtained through a
computationally efficient time-stepping iterative solution. This

breach erosion model was *not* subject to numerical stability/
convergence difficulties experienced by the more complex model of
Ponce and Tsivoglou(1981). The model's predictions were
favorably compared with observations of a piping failure of the
large man-made Teton Dam in Idaho, the piping failure of the
small man-made Lawn Lake Dam in Colorado, and an overtopping
activated breach of a large landslide-formed dam in Peru. Model
sensitivity to numerical parameters was minimal. A variation of
± 30 percent in the internal friction angle and a ± 100 percent
variation in the cohesion parameter resulted in less than ± 20
percent variation in the simulated breach properties and peak
breach outflow. However, it was somewhat sensitive to the extent
of grass cover when simulating man-made dams in which over
topping flows could or could not initiate the failure of the dam.

A brief description of three breach simulations follows:

(1) Teton Dam. The BREACH model was applied to the piping
initiated failure (Fread;1984,1987) of the earthfill Teton Dam
which breached in June 1976, releasing an estimated peak
discharge (Q_p) of 2.2 million cfs having a range of 1.6 to 2.6
million cfs. The material properties of the breach were as
follows: H_d=262.5 ft, D_{50}=0.1mm, ϕ=20 deg, C_h=30 lb/ft⁻, and γ=100
lb/ft³. The downstream face of the dam had a slope of 1: 4 and
upstream face slope was 1:2. An initial piping failure of 0.01
ft located at 160 ft above the bottom of the dam commenced the
simulation. The simulated breach hydrograph is shown in
Figure 5. The computed final breach top width (W) of 645 ft
compared well with the observed value of 650 ft. The computed
side slope of the breach was 1:1.06 compared to 1:1.0. The⁻
computed time (T_p) to peak flow was 2.2 hr compared to 1.95-2.12
hr.

(2) Lawn Lake Dam. This dam was a 26 ft high earthen dam with
approximately 800 acre-ft of storage, which failed July 15, 1982,
by piping along a bottom drain pipe (Jarrett and Costa,1984).
The BREACH model was applied (Fread,1987) with the piping breach
assumed to commence within 2 ft of the bottom of the dam. The
material properties of the breach were assumed as follows:
H_d=26 ft, D_{50}=0.25mm, ϕ=25 deg, C_h=100 lb/ft⁻, and γ=100 lb/ft³.
The downstream face of the dam had a slope of 1:3 and the
upstream face 1:1.5. The computed outflow was 17,925 cfs, while
the estimated actual outflow was 18,000 cfs. The model produced
a trapezoidal-shaped breach with top and bottom dimensions of 132
and 68 ft, respectively. The actual breach dimensions were 97
and 55 ft, respectively. The mean observed breach width was
about 32 percent smaller than the mean breach width produced by
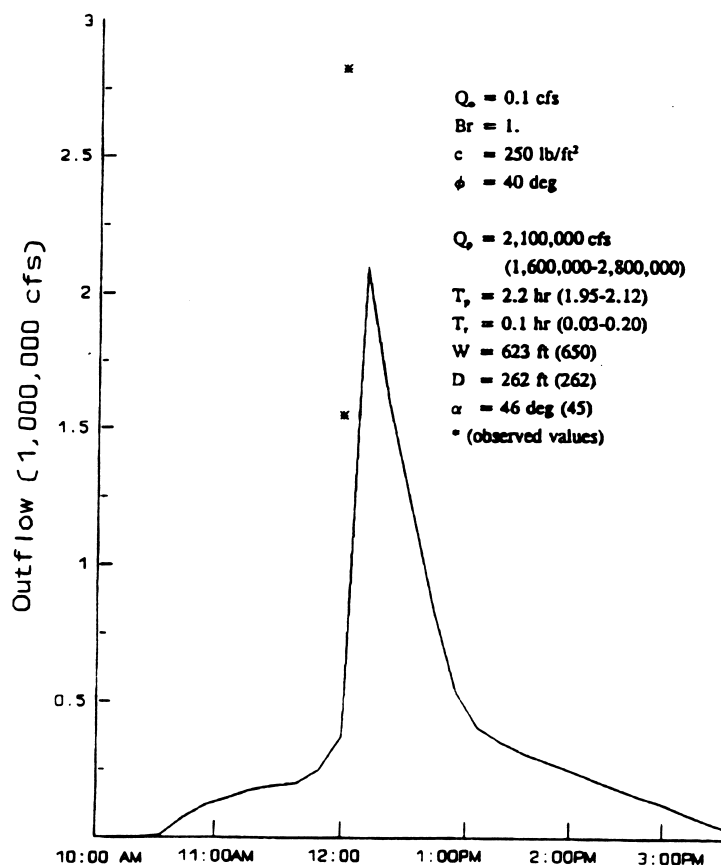the model.

Figure 5. Teton Dam: Predicted and Observed Breach Outflow
Hydrograph and Breach Properties

(3)   Mantaro Landslide Dam.   A massive landslide occurred in the
valley of the Mantaro River in the mountainous area of central
Peru on April 25, 1974. The slide, with a volume of
approximately 5.6 x 10$^{\cdot \cdot}$ft, dammed the Mantaro River and formed a
lake which reached a depth of about 560 ft before overtopping
during the period June 6-8, 1974 (Lee and Duncan,1975). The
overtopping flow very gradually eroded a small channel along the
approximately 1 mile long downstream face of the slide during the
first two days of overtopping. Then a dramatic increase in the
breach channel occurred during the next 6-10 hours resulting in a
final trapezoidal-shaped breach channel approximately 350 ft
deep, a top width of some 800 ft, and side slopes of about 1:1.
The peak flow was estimated at 353,000 cfs as reported by Lee and
Duncan(1975), although Ponce and Tsivoglou(1981) later reported
an estimated value of 484,000 cfs. The breach did not erode down
to the original river bed; this caused a rather large lake about
200 ft deep to remain after the breaching had subsided some 24
hours after the peak had occurred. The landslide material was
mostly a mixture of silty sand with some clay resulting in a D_{50}
size of about 11mm with some material ranging in size up to 3 ft
boulders. The BREACH model was applied (Fread;1984,1987) to the
Mantaro landslide-formed dam using the following parameters:

upstream face slope 1:17, downstream face slope 1:8, H_d=560 *ft*, D_{50}=11mm, C_h=30 lb/ft^2, ϕ=38 deg, γ=100 lb/ft^3. The initial breach depth was assumed to be 0.35 ft. The computed breach outflow is shown in Figure 6 along with the estimated actual values. The timing of the peak outflow and its magnitude are very similar as are the dimensions of the gorge eroded through the dam shown by the values of *D*, *W*, and α in Figure 6. Of particular interest, the BREACH model produced a depth of breach of 352 ft which compared to the observed depth of 350 ft.
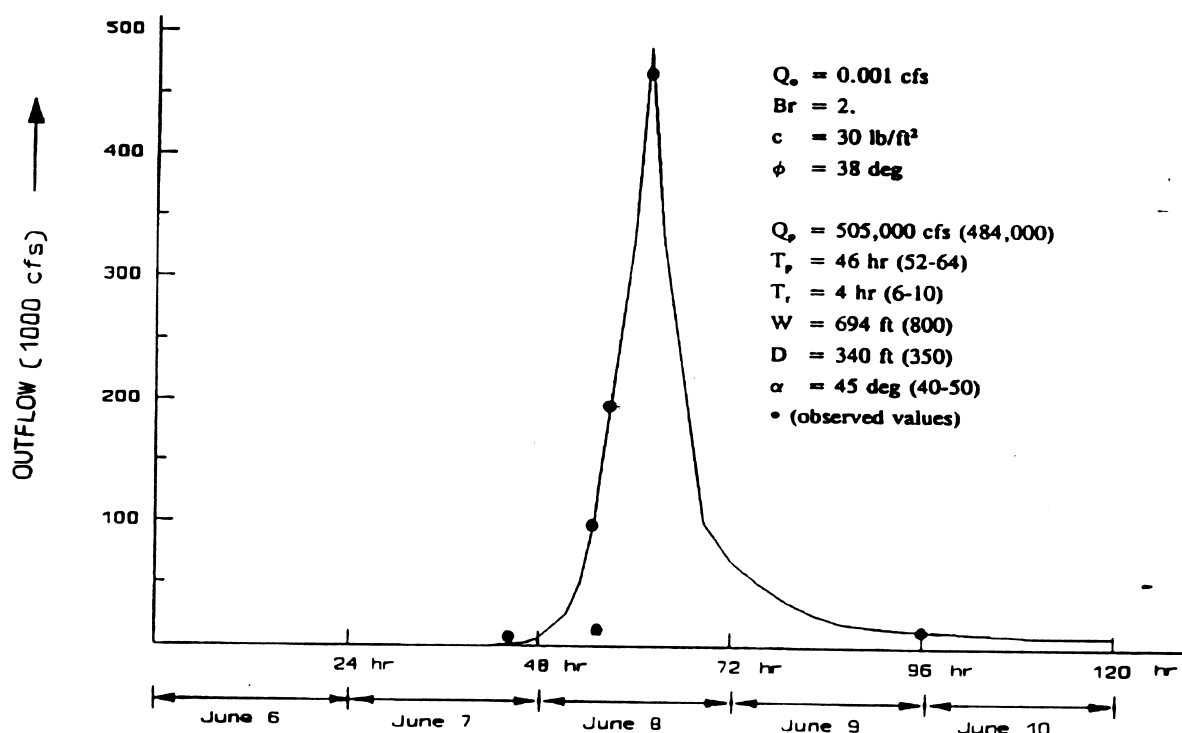

Figure 6. Mantaro Landslide Dam: Predicted and Observed Breach Hydrograph and Breach Properties.

Other physically-based breach erosion models include the following: (1) the BEED model (Singh and Quiroga,1988) which is similar to the BREACH model except it considers the effect of saturated soil in the collapse of the breach sides and it routes the breach outflow hydrograph through the downstream valley using a simple diffusion routing technique (Muskingum-Cunge) which neglects backwater effects and can produce significant errors in routing a dam-breach hydograph when the channel/valley slope is less than 0.003 ft/ft; (2) a numerical model (Macchione and Sirangelo,1988) based on the coupling of the one-dimensional unsteady flow (Saint-Venant) equations with the continuity equation for sediment transport and the Meyer-Peter and Muller sediment transport equation; and (3) a numerical model (Bechteler and Broich,1993) based on the coupling of the two-dimensional

unsteady flow equations with the sediment continuity equation and the Meyer-Peter and Muller equation.

4. Flood Routing

Flood waves produced by the breaching (failure) of a dam are known as dam-breach flood waves. They are much larger in peak magnitude, considerably more sharp-peaked, and generally of much shorter duration with flow acceleration components of a far greater significance than flood waves produced by precipitation runoff. The prediction of the time of occurrence and extent of flooding in the downstream valley is known as flood routing. The dam-breach wave is modified (attenuated, lagged, and distorted) as it flows (is routed) through the downstream valley due to the effects of valley storage, frictional resistance to flow, flood flow acceleration components, flow losses, and downstream channel constrictions and/or flow control structures. Modifications to the dam-break flood wave are manifested as - attenuation (reduction) of the flood peak magnitude, spreading-out or dispersion of the temporal varying flood-wave volume, and changes in the celerity (propagation speed) or travel time of the flood wave. If the downstream valley contains significant storage volume such as a wide floodplain, the flood wave can be extensively attenuated (see Figure 7) and its propagation speed greatly reduced. Even when the downstream valley approaches that of a relatively narrow uniform rectangular-shaped section, there is appreciable attenuation of the flood peak and reduction in the wave celerity as the wave progresses through the valley.
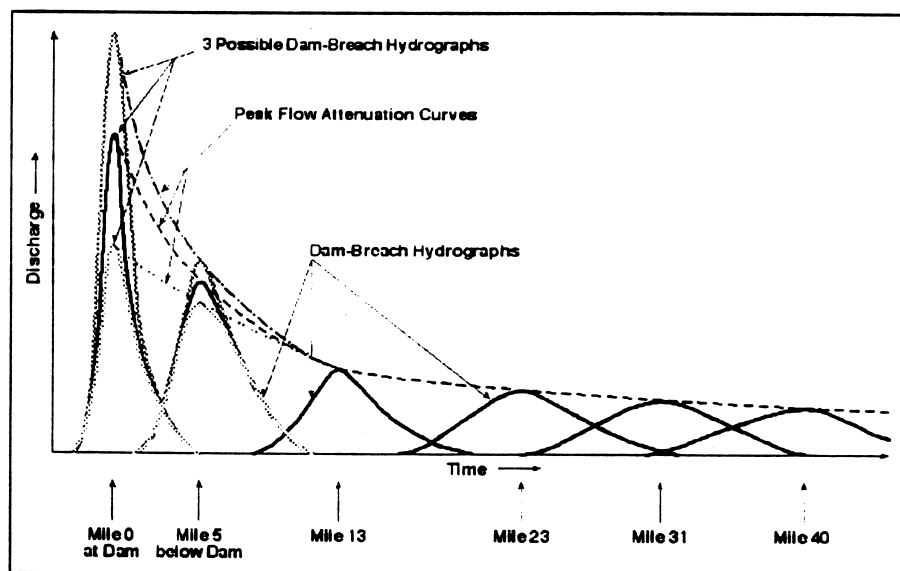

Figure 7. Dam-Break Flood Wave Attenuation Along the Routing Reach.

5.1 Flood Routing with Saint-Venant Equations

Dam-breach flood waves have been routed using simulation models
based on numerical solutions of the one-dimensional Saint-Venant
equations of unsteady flow, e.g., DAMBRK (Fread,1977,1978) and
FLDWAV (Fread,1993). The Saint-Venant equations used in these
models consists of the mass conservation equation, i.e.,

$$\partial Q/\partial x + s_c \partial (A + A_o)/\partial t - q = 0 \tag{8}$$

and the momentum conservation equation, i.e.,

$$s_m \partial Q/\partial t + \beta(\partial Q^2/A)/\partial x + gA(\partial h/\partial x + S_f + S_e) - qv_x = 0 \tag{9}$$

where h is the water-surface elevation, A is the active cross-
sectional area of flow, A_o is the inactive (*off-channel storage*)
cross-sectional area, s_c and s_m are depth-weighted sinuosity
coefficients which correct for the departure of a sinuous in-bank
channel from the x-axis of the valley floodplain, x is the
longitudinal mean flow-path distance measured along the center of
the river/valley watercourse (river channel and floodplain), t is
time, q is the lateral inflow or outflow per lineal distance
along the river/valley (inflow is positive and outflow is
negative), β is the momentum coefficient for nonuniform velocity
distribution within the cross section, g is the gravity
acceleration constant, S_f is the boundary friction slope, S_e is
the expansion-contraction (large eddy loss) slope, and v is the
velocity component of the lateral flow along the x-axis.

5.2 Peak Flow Routing Attenuation Curves

Another flood routing technique SMPDBK (Fread and Wetmore,1984;
Fread, et al.,1991) has been used when the river/valley
downstream from a breached dam is uncomplicated by unsteady
backwater effects, levee overtopping, or large tributaries.
SMPDBK determines the peak flow, depth, and time of occurrence at
selected locations downstream of a breached dam. SMPDBK first
computes the peak outflow at the dam, based on the reservoir size
and the temporal and geometrical description of the breach. The
SMPDBK uses an analytical time-dpendent broad-crested weir
equation, Eq.(6), to determine the maximum breach outflow (Q_p) in
cfs and the user is required to supply the values of four
variables for this equation. These variables are: (1) the
surface area (A_s, acres) of the reservoir; (2) the depth (H_d, ft)
to which the breach erodes; (3) the time (t_f, hrs) required for
breach formation; and (4) the width (b_{av}, ft) of the breach, and
(5) the spillway flow and overtopping crest flow (Q_o) which is
estimated to occur simultaneously with the breach peak outflow.
The computed flood wave and channel properties are used in
conjunction with special dimensionless routing curves (see
Figure 8) to determine how the peak flow will be diminished as it
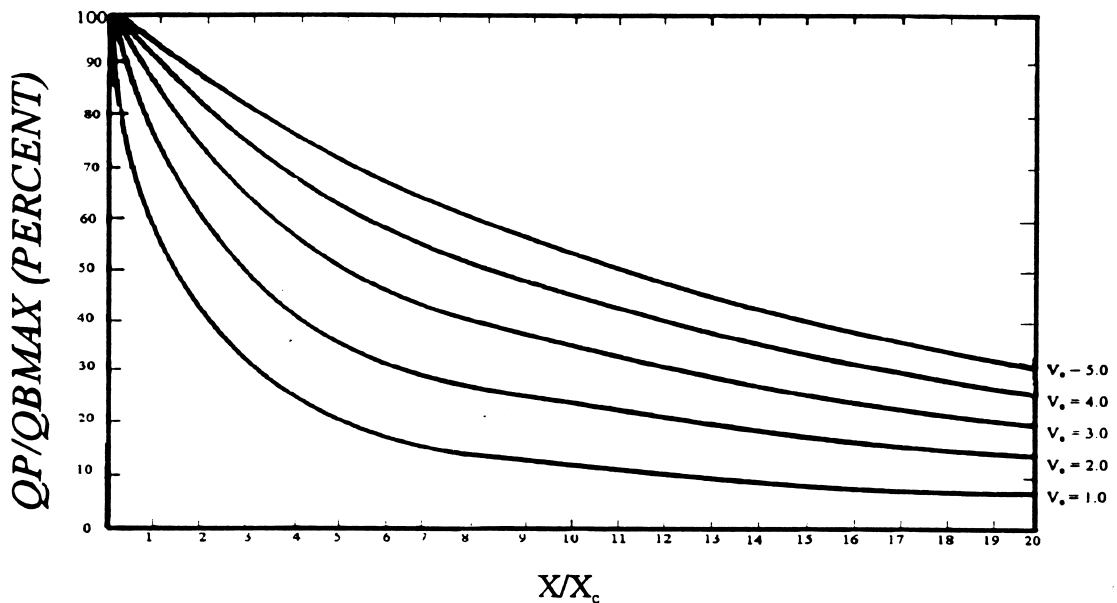moves downstream.

Figure 8. Routing Curves for SMPDBK Model for Froude No. = 0.25.

The dimensionless routing curves were developed from numerous executions of the NWS DAMBRK model and they are grouped into families based on the Froude number associated with the flood wave peak, and have as their X-abcissa the ratio of downstream distance (from the dam to a selected cross-section where Q_p and other properties of the flood wave are desired) to a distance parameter (X_c). The Y-ordinate of the curves used in predicting peak downstream flows is the ratio of the peak flow (Q_p) at the selected cross section to the computed peak flow at the dam, QBMAX. The distinguishing characteristic of each member of⁻a family is the ratio (V_o) of the volume in the reservoir to the average flow volume in the downstream channel from the dam to the selected section. To specify the distance in dimensionless form, the distance parameter $(X.)$ in feet is computed as follows:

$$X_c = 6V_r/[(1 + 4(0.5)^{m-1}A_d]$$ (10)

in which V_r is the reservoir volume (acre-ft), m is a cross-sectional shape factor for the routing reach, and A_d is the average cross-section area in the routing reach at a depth of H_d. The volume parameter (V_o) is $V_o=V_r/(\bar{A}_c X_c)$ in which $\bar{A}_c$ represents the average cross-sectional area in the routing reach at the average maximum depth produced by the routed flow. The Froude Number (F_c) is $F_c=V_c/(gD_c)$ where V_c and D_c are the average velocity and hydraulic depth, respectively, within the routing reach. Further details on the computation of the dimensionless parameters can be found elsewhere (Wetmore and Fread,1984; Fread,et al.,1991).

The SMPDBK model then computes the depth produced by the peak flow using the Manning equation based on the channel geometry, slope, and roughness at the selected downstream locations. The model also computes the time required for the peak to reach each

forecast location and, if a flood depth is entered for the point, the time at which that depth is reached, as well as when the flood wave recedes below that depth, thus providing a time frame for evacuation and possible fortification on which a preparedness plan may be based. The SMPDBK model neglects backwater effects created by any downstream dams or bridge embankments, the presence of which may substantially reduce the model's accuracy. However, its speed and ease of use, together with its small computational requirements, make it an attractive tool for use in cases where limited time and resources preclude the use of the DAMBRK or FLDWAV models. In such instances, planners, designers, emergency managers, and consulting engineers responsible for predicting the potential effects of a dam failure may employ the model where backwater effects are not significant.

The SMPDBK model was compared with the DAMBRK model in several theoretical applications (Fread, et al.,1991) and several hypothetical dam failures (Westphal and Thompson,1987) where the effects of backwater, downstream dams/bridges, levee overtopping, or significant downstream tributaries were negligible. The average differences between the two models were less than 10 percent for predicted flows, travel times, and depths.

5.3 Numerical Routing with Muskingum-Cunge Equation

Another simple routing model (Muskingum-Cunge with variable coefficients) may be used for routing dam-breach floods through downstream river/valleys with moderate to steep bottom slopes ($S_o > 0.003$ ft/ft). The spatially distributed Muskingum-Cunge routing equation applicable to each Δx_i subreach for each Δt^+ time step is as follows:

$$Q_{i+1}^{j+1} = C_1 Q_i^{j+1} + C_2 Q_i^j + C_3 Q_{i+1}^j + C_4 \tag{11}$$

The coefficients C_1, C_2, and C_3 are positive values whose sum must equal unity; they are defined as

$$C_o = 2 K(1-X) + \Delta t \tag{12}$$

$$C_1 = (\Delta t - 2KX)/C_o \tag{13}$$

$$C_2 = (\Delta t + 2KX)/C_o \tag{14}$$

$$C_3 = [2K(1-X) - \Delta t]/C_o \tag{15}$$

$$C_4 = \overline{q}_i \, \Delta x \, \Delta t / C_o \tag{16}$$

in which K is a storage constant having dimensions of time, X is a weighting factor expressing the relative importance of inflow and outflow on the storage in the Δx_i subreach of the river, and q_i the lateral inflow (+) or outflow (−) along the Δx_i subreach.

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K and *X* are computed as follows:

$$K = \Delta x_i / c \tag{17}$$

$$X = 0.5[1-D/(k'S\Delta x_i)] \tag{18}$$

in which *c* is the kinematic wave celerity $c=k'Q/A$, *Q* is discharge, *S* is the energy slope approximated by evaluating S_f for the initial flow condition, *D* is the hydraulic depth *A/B* where *A* is the cross-sectional area and B is the wetted top width associated with *Q*, and *k'* is the kinematic wave ratio, i.e., $k'=5/3-2/3$ *A (dB/dh)/B*. The bar indicates the variable is averaged over the Δx_i reach and over the Δt^j time step. The coefficients $(C_0, C_1, C_2, C_3, C_4)$ are functions of Δx_i and Δt^j (the independent parameters), and *D, c,* and *k'* (the dependent variables) are also functions of water-surface elevations(*h*).

These water-surface elevations may be obtained from a steady, uniform flow formula such as the Manning equation, i.e.,

$$Q = \mu/n \; A \; R^{\frac{2}{3}} \; S^{\frac{1}{2}} \tag{19}$$

in which *n* is the Manning roughness coefficient, A is the cross-sectional area, *R* is the hydraulic radius given by *A/P* in which *P* is the wetted perimeter of the cross section, S is the energy slope as defined previously, and μ is a units conversion factor (1.49 for U.S. and 1.0 for SI).

5.4 Testing of DAMBRK and SMPDBK

The DAMBRK and SMPDBK models have been tested on several historical floods due to breached dams to determine their ability to reconstitute observed downstream peak stages, discharges, and travel times. Among the floods that have been used in the testing are: 1976 Teton Dam Flood, 1972 Buffalo Creek (coal-waste dam) Flood, 1889 Johnstown Dam Flood, 1977 Toccoa (Kelly Barnes) Dam Flood, the 1997 Laurel Run Dam Flood and others. Some of the results from the Teton and Buffalo Creek dam-breach tests follow.

The Teton Dam, a 300 ft high earthen dam with 230,000 acre-ft of stored water and maximum 262.5 ft water depth, failed on June 5, 1976, killing 11 people making 25,000 homeless and inflicting about $400 million in damages to the downstream Teton-Snake River Valley. The following observations were reported (Ray, et al.,1976): the approximate development of the breach, description of the reservoir storage, downstream cross-sections and estimates of Manning's n approximately every five miles, estimated peak discharge measurements of four sites, flood-peak travel times, and flood-peak elevations. The critical breach parameters were $t_f=1.43$ hours, *b*=80 ft, and *z*=1.04. The computed peak flow profile along the downstream valley is shown in Figure 9. Variations between computed and observed values are

about 5 percent for DAMBRK and 12 percent for SMPDBK. The
Buffalo Creek "coal waste" dam, a 44 ft high tailings dam with
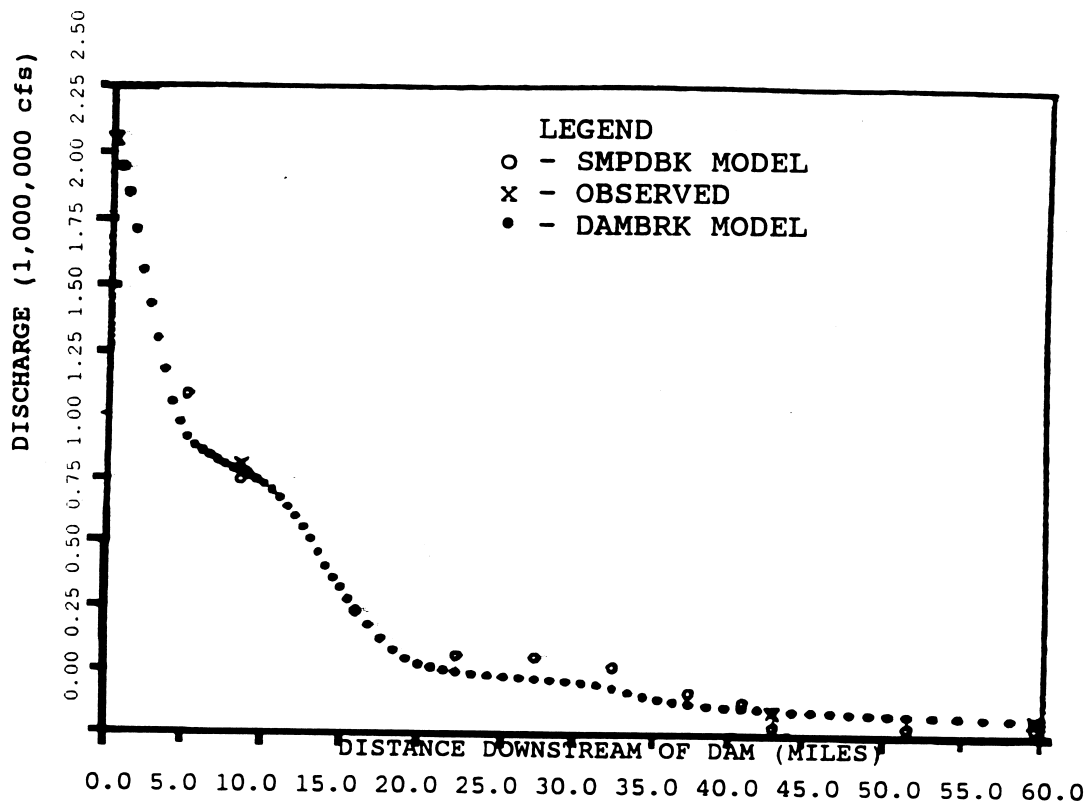400 acre-ft of storage failed on February 1972, resulting in

Figure 9. Profile of peak discharge downstream of Teton.

118 lives lost and over $50 million in property damage. Flood
observations (Davies, et al., 1975) along with the computed
flood-peak profile extending about 16 miles downstream are shown
in Figure 10. Critical breach parameters were t_f=0.08 hours,
b=170 ft, and z=2.6. Comparison of computed and observed flows
indicate an average difference of about 11 percent for both
DAMBRK and SMPDBK.

The Muskingum-Cunge flood routing model was compared with the
DAMBRK (Saint-Venant) model for all types of flood waves (Fread
and Hsu,1993). For dam-breach waves, the routing error
associated with the more simple but less accurate Muskingum-Cunge
model was found to exceed 10 percent when the channel bottom
slope $S_o < 0.004/t_f^{0.89}$; the error increased as the bottom slope
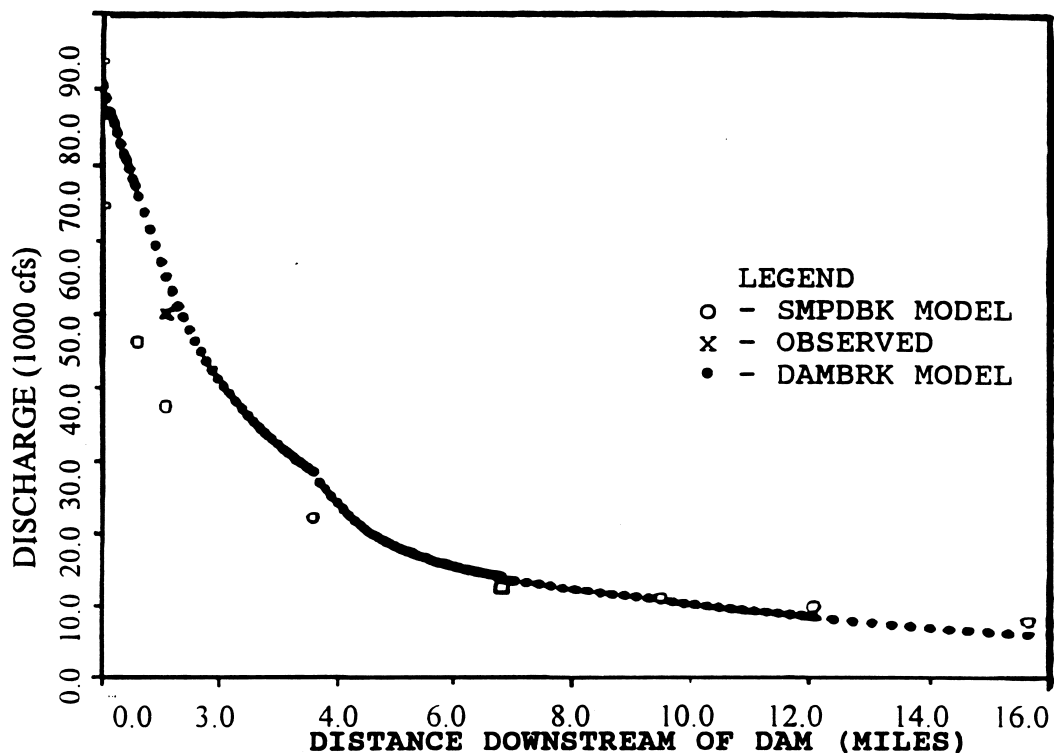became more mild and as the time of failure (t_f) became smaller.

Figure 10. Profile of peak discharge downstream of Buffalo Creek

5.   Future Research/Development Directions

Future research/development directions that are judged to mÖst
efficiently and effectively improve the prediction capabilities
for dam-breach floods are the following in order of priority: (1)
use prototype physical experiments to develop breach models for
embankment dams including the complexity of both breach
"initiation" time and "formation" time; first, for clay
embankment dams (Temple and Moore,1997) but also, for silt/loam
embankments, sand/gravel embankments, and embankments with clay
or concrete seepage-prevention cores; (2) determine the Manning n
flow resistance values for dam-breach floods using both
historical data from such floods and theoretical approaches,
e.g., physically-based friction component analysis similar to
that used by Walton and Christianson(1980) which was patterned
after the Colebrook friction equation (Streeter,1966). Also,
determine procedures to account for flood debris blockage effects
on Manning n values and the damming effect on bridge openings;
and (3) develop methodologies, e.g., Monte-Carlo simulation
(Froehlich,1998), to produce the inherent probabilistic features
of dam-breach flooding due to uncertainties in reservoir inflows,
breach formation, and downstream Manning n/debris effects.

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